CONCRETE AND CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

MAY 1959

VOL. LIV. NO. 5

FIFTY-FOURTH YEAR OF PUBLICATION

PRICE 2s. ANNUAL SUBSCRIPTION 24s. POST FREE. $3.90 in Canada and U.S.A.

LEADING CONTENTS

No. 618

ISSUED MONTHLY

Registered for Canadian Magazine Post

INDEX OF ADVERTISERS

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

Volume LIV, No. 5. LONDON, MAY, 1959.

EDITORIAL NOTES

Architects and Engineers.

IN ancient times buildings of architectural merit were the result of one man applying his knowledge of architecture and the mechanics of construction and his ability to control craftsmen and to do the work now delegated to a quantity surveyor; he was in fact the master of all the works, as were Rennie, Telford, Brunel and others in more recent years. With the introduction into building of iron and steel, and more recently of reinforced and prestressed concrete, aluminium, and other new materials, and with modern complicated systems of heating, ventilation, plumbing, and so on, the architect has had more and more to delegate structural and other design work to engineers and specialists, whose suggestions may influence the architect's design.

When an architect is appointed to design an essentially engineering structure he employs an engineer to design it, and if the engineering design is a sound one that makes the best use of materials there is little the architect can do to alter it without adding to the cost. It has been said that in such cases the contribution of the architect is only that of a dresser, and if there is any truth in the view so often held by architects that a structure should express its construction then the work of the architect can actually detract from its architectural merit. A comment on the architectural merit of many structures in the style that is called contemporary since ultra-modern became unfashionable is given by Mr. G. J. Howling in an article in "The Builder" on the work of the late Sir Edwin Lutyens: " It is interesting ", he writes, " to speculate on what Lutyens would have done if he had practised in these days of the glass cage, standardisation, mass production, and ruthless economy—perhaps he would have renounced architecture."

In these circumstances it is not surprising that the relative positions of architect, engineer, and contractor have been the subject of public argument. A discussion at the Royal Institute of British Architects on the employment of architects by contractors was reported in this journal for January last. Although they heartily objected to this practice the members present decided that they could not stop it and that it would probably be unwise to try to do so. No doubt they had in mind the discussions of thirty or forty years ago when objection was taken to the setting up of architectural departments by local authorities, and more recently to this practice being followed by nationalised industries and large industrial concerns. On these occasions no action was taken or could have

been taken, and we have since seen the employment of "official" architects grow until more than half the members of the Institute are now so employed.

In January last a joint meeting was convened by the Royal Institute of British Architects and the Institution of Civil Engineers to discuss the relationship between architects and engineers, and some strong opinions were expressed. Engineers were aggrieved that architects should be designated master builders of structures that are essentially matters of engineering about which the architect may know little or nothing and who can make suggestions only on questions of lines and decoration. It was thought that in such cases the engineer should be designated the master builder, and the architect be paid a lump-sum fee for his services. Some of the architects present were no doubt anxious about their increasing need to employ specialists because of their own lack of knowledge, their contribution being often restricted to the production of plans and elevations conforming to the views of the engineer and contractor and making business arrangements between the owner of the building, the various consultants, the main contractor, and sub-contractors. This is vastly different from the original conception of the master builder, and it may sometimes be that by reason of his training the architect is the least qualified of the team to deal with commercial affairs. Nowadays most new buildings are plain engineering structures in which the only contribution of the architect seems to be the plan and the decision to use one of the ideas recently imported from abroad, such as giving a building the appearance of being propped on stilts, the use of large precast members, "shell" roofs, horizontality, verticality, and so on. Such buildings depend for their appearance more on the mathematics of engineering than the art of architecture. This is a result of the utilitarian age in which we live and the changing ideas of the proper use of capital. No longer are churches built to the glory of God, or office buildings to do honour to the businesses that occupy them. Churches are now built to resemble places of entertainment rather than worship, and wealthy businesses prefer to rent buildings erected by speculators whose only purpose is to obtain the greatest rentable floor space at the least cost. It is true that attempts are often made to give commercial buildings what architects call "structural expression", but this seems to be only what engineers call structural design.

The chairman of the meeting was Mr. Basil Spence, President of the Royal Institute of British Architects, who said that when he was a professor of architecture he found that students in the engineering department had no respect whatever for architects, whereas the students of architecture respected engineers. This may derive from the engineer's knowledge that he can design a complicated structure without the aid of an architect, whereas few architects to-day could design such a structure of which they had drawn the plans and elevations. Suggestions were made that architects should be taught more structural engineering and that engineers should be taught architecture. This might result in a body of architect-engineers who could completely design a structure, and who would indeed be master builders. Some such men already exist in most countries, but generally they are engineers who have studied architecture or have natural gifts rather than architects who have studied engineering. The plan of the structure is the basis of architecture, and it would be useful if engineers were taught planning at the beginning of their training instead of, as is now generally the case, at the end.

Losses of Prestressing Force.

Saving Possible by Exact Determination.

By Dr.-Ing. JERZY ZIELINSKI (WARSAW).

In prestressed concrete structures it is important to determine the actual prestressing force as precisely as possible, as differences in amount and position between the actual and assumed forces may lead to different stresses and deformations from those anticipated.

The analysis of errors which occur in the determination of the prestressing force constitutes the basis of any design from statistical data. This method requires a knowledge not only of minimal values of dimensions, strengths of materials, and prestressing forces, but also of the deviations from these values. The results of such an analysis by the Swedish Building Research Station, when applied to the design of floors about 11,000 sq. yd. in area, resulted in a saving of about 30 per cent. of the steel calculated by normal methods.

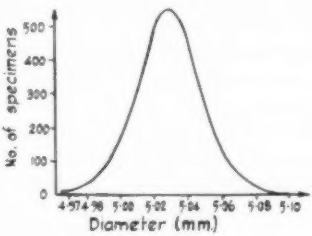


Fig. 1.

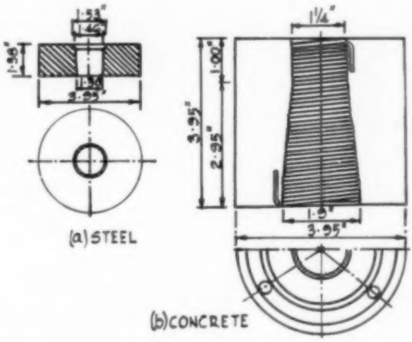


Fig. 2.

The changes of prestressing force due to the properties of the steel and concrete have been investigated in many laboratories, but the losses, and errors in determining the losses, of prestressing force during tensioning until the cable is anchored—with particular regard to straight cables—have not yet been determined completely, although Mr. Cooley and M. Montagnon have pointed out their importance and have done much valuable work on this subject.

The tests and analyses described in the following apply to Freyssinet cables as used in Poland, tensioned by double-acting jacks and anchored by reinforced concrete or steel cones.

Analysis of Errors and Losses.

(1) Tolerances on Diameters of Wires.—The tolerances permitted by various national standards vary slightly, and lead to expected errors in the prestressing force of between -3 and $+4$ per cent., typical values being $\pm 2{\cdot}4$ per cent. (Great Britain), ± 3 per cent. (U.S.A.), -2 to $+4$ per cent. (Netherlands), $+4$ per cent. (Italy), and $\pm 2{\cdot}4$ per cent. (Poland).

A statistical analysis of 2542 specimens of 5-mm. wire, with a tensile strength

of 235,000 lb. per square inch, used in Poland for post-tensioned cables, showed that the mean diameter was not 5·00 mm. but 5·03 mm. (*Fig.* 1), and the probability that the diameter of any wire is within the standard tolerance of $\pm 0{\cdot}06$ mm. of the nominal diameter of 5·00 mm. is 0·933. The probability that it is within $\pm 0{\cdot}06$ mm. of the mean value of 5·03 mm. is 0·9973. The probable errors in assessing the prestressing force are therefore not $\pm 2{\cdot}4$ per cent. (corresponding to the range $4{\cdot}94 < d < 5{\cdot}06$) but $-1{\cdot}2$ to $+3{\cdot}7$ per cent. (corresponding to the range $4{\cdot}97 < d < 5{\cdot}09$). The confidence limit* of 99 per cent. corresponding to the mean diameter of 5·03 mm. and the calculated standard deviation of 0·02 mm. is $5{\cdot}02 < d < 5{\cdot}04$, resulting in probable errors of $\pm 0{\cdot}4$ per cent. in the assessment of the prestressing force.

(2) LOSSES IN THE JACKS.—These losses are caused by friction between the pistons and the washers and by the resistance of the springs. For a long time these losses were ignored; even now they are often neglected, or included with the losses due to friction at the anchors. Tests were made on twenty double-acting jacks of two types produced in Poland and the results analysed statistically in order to determine whether all jacks of the same type had the same losses, and whether the losses were constant for all ranges of pressure. The analysis showed that the losses were constant for jacks of type I for a range of pressure from 0 to 250 atmospheres, and were between 2 and 12 per cent. The losses in two jacks of the same type may therefore differ by 10 per cent.

The assumption of a single mean value for the internal losses at the jacks, for example 9 per cent., would lead to errors in assessing the prestressing force of, say, -3 to $+7$ per cent. This method is inaccurate, and therefore all jacks should be tested to determine their characteristics.

(3) LOSSES DUE TO FRICTION AT THE ANCHORS.—These losses were ascertained for the steel and reinforced concrete anchors shown in *Fig.* 2, using two types of jacks. It was found that the losses due to friction of wires in the anchors depend partly on the type of jack and partly on the type of anchor. With one type of steel anchor the losses depend on the hardness of the steel.

The confidence limits of 99 per cent. for the losses μ of prestressing force caused by friction at the anchors were as follows.

Jacks Type I.—Steel anchor-plates, Rockwell "C" Hardness No. 45, $1{\cdot}27 < \mu < 1{\cdot}73$ per cent. Reinforced concrete female cones and steel male cones, $0{\cdot}84 < \mu < 1{\cdot}16$ per cent.

Jacks Type II.—Steel anchor-plates, Rockwell "C" Hardness No. 45, $5{\cdot}82 < \mu < 6{\cdot}58$ per cent. Steel anchor plates, Brinell Hardness No. 250 (Rockwell "C" Hardness No. 25), $4{\cdot}13 < \mu < 4{\cdot}87$ per cent. Reinforced concrete female cones and steel male cones, $3{\cdot}04 < \mu < 3{\cdot}50$ per cent.

The greatest difference between the mean value and the upper and lower values of the confidence limit is 0·48, which produces an error of 0·4 per cent. in the assessment of the prestressing force in a cable with twelve 5-mm. wires and a stress of 150,000 lb. per square inch. If the loss is ignored, the error is about 5 per cent. *Fig.* 3 shows the angles of the wires between the anchors and the jacks.

(4) LOSSES DUE TO SLIPPING.—These losses are caused when the temporary

* The confidence limit is the range within which the true value of the mean (as distinct from the value determined for the sample) can be placed with a certain assumed probability (for example, 0·99).

anchorage of the cable on the jack is replaced by the permanent anchorage at the end of the member. The amount of slipping depends on two groups of factors. The first comprises factors which depend on the size of the jack and the hardness of the anchors; the second comprises such accidental factors as the tolerances on the wires and the anchoring equipment and the state of the surfaces of the wires.

Investigations were made at nine structures under normal working conditions, and included slipping of single wires in addition to the normal slipping of all wires at an anchor. A total of 1640 cables, with lengths between 43 ft. and 210 ft.,

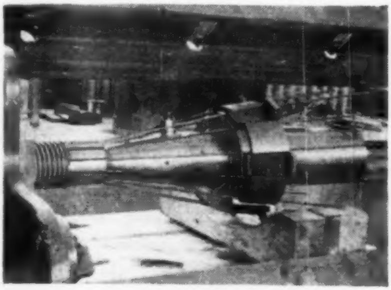


Fig. 3.—Inclination of Wires at Jack.

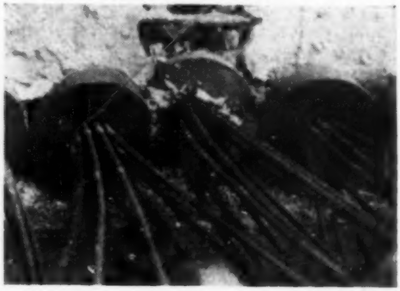


Fig. 4.—Partial Failure of Concrete at Anchorage.

were inspected; 1194 of these had steel anchors and 446 had reinforced concrete anchors, steel male cones being used in all cases.

The mean values and standard deviations of the amount of slipping were obtained for each structure, and confidence limits of 99 per cent. were determined. It was found that the magnitude of the slipping is independent of the type of jack (possibly because the differences in the anchoring forces were very small), but is affected by the type of anchor. The confidence limits of 99 per cent. for the slip p for different types of anchor are as follows. Steel anchor plates, Rockwell "C" Hardness No. 45, $4{\cdot}25 < p < 5{\cdot}20$ mm. Steel anchor-plates, Brinell Hardness No. 250 (Rockwell "C" Hardness No. 25), $3{\cdot}11 < p < 3{\cdot}39$ mm. Reinforced concrete female cones and steel male cones (Rockwell "C" Hardness No. 40), $3{\cdot}69 < p < 4{\cdot}29$ mm.

The errors in assessing the prestressing force which occur when slipping is not taken into account depend on the length of the cable. For cables 30 ft., 60 ft., and 90 ft. long, tensioned from one end, the errors would be about 8, 4, and 2 per cent. respectively. The greatest difference between the upper and lower confidence limits and the mean value of the slipping is 0·3 mm. in a cable 30 ft. long with a stress of 150,000 lb. per square inch; the resulting error would be 0·6 per cent.

Slipping of single wires occurred in the case of 1·6 per cent. of the wires, and the mean value of the loss of prestressing force was about 0·6 per cent. These figures do not include cases in which slipping of all the wires occurred as a result of insufficient hardness of the steel plate.

(5) LOSSES DUE TO DEFORMATION OF THE CONCRETE UNDER THE ANCHOR AND BREAKING OF SINGLE WIRES.—These losses are rare. Of 3308 cables examined only 11 cases of deformation were observed, that is 0·3 per cent.; the loss of prestressing force so caused was about 20 per cent. of the initial force in the cable, so that the total loss of prestressing force due to deformation was about 0·06 per cent. *Fig.* 4 shows an example of this.

The breaking of a single wire in a cable gives rise to a similar loss.

(6) LOSSES DUE TO INACCURATE MEASUREMENT.—The mean values of the errors in assessing the prestressing force due to mistakes in readings were between ±3·4 per cent.

(7) LOSSES DUE TO FRICTION IN STRAIGHT CABLES.—Until recently it was considered that losses due to friction were of importance only in curved cables; losses in cables which are nominally straight were ignored or treated as accidental. The importance of losses in straight cables is now acknowledged, and tests have been made in sheet-metal ducts under normal working conditions, with the object of (1) determining whether the distribution of the losses along the length of the cable is random or regular, (2) establishing mean values and confidence limits for the losses per unit length, and the consequent errors in the assessment of the prestressing force, and (3) expressing the change in the prestressing force as a function of the length of a straight cable.

The force at the end of the cable opposite to the tensioning equipment was measured by means of a mechanical dynamometer connected to the cable by the arrangement shown in *Fig.* 5. Measurements were also made at openings along the beam by means of inductional tensiometers (*Fig.* 6). The tests were made on 1100 cables at six structures, and included (1) direct measurement of the prestressing force at each end of the cable, at intervals of about 2·5 tons, together with the corresponding elongations, and (2) indirect determination of the prestressing force at intervals along the length by means of elongations measured at the openings.

The mean values of the elongations, the standard deviations, and confidence limits of 99 per cent. were calculated. It was found that the function describing

Fig. 5.—Dynamometer in Use.

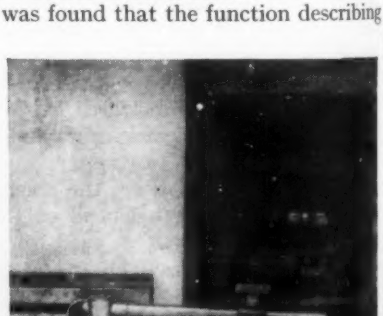


Fig. 6.—Inductional Tensiometer in Position.

the changes of prestressing force along the length l was linear, and hence the prestressing force S_w calculated from the elongation could be expressed as $S_w = K.S_m$, in which K is a coefficient depending on the length, losses per unit length, and method of tensioning, and S_m is the tensioning force based on the pressure at the jack. S_w may be considered as a mean value of S_m at the beginning of the cable and S_k the mean value at the end. Hence $S_w = \frac{S_m + S_k}{2}$. Expressing S_k as a function of losses λ per unit length of the cable, then $S_w = S_m\left(1 - \frac{l\lambda}{2}\right)$.

Hence $K = 1 - \frac{l\lambda}{2}$ and $S_k = S_m - (1 - l\lambda)$.

Confidence limits of 99 per cent. for K were calculated, and the mean values of losses per unit length, together with their confidence limits, were calculated.

TABLE I.

Method of forming ducts	Number of cables tested	Mean value of loss per unit length (per cent.)	99 per cent. confidence limit (per cent.)
Metal tube withdrawn before concrete hardens .	43	0·00	0·00–0·15
Rubber tube filled with water under pressure .	217	0·62	0·51–0·73
Cable in bituminous paper tube . . .	35	1·29	1·16–1·42
Metal sheet 0·3 mm. thick	74	0·59	0·41–0·77
Metal sheet 0·6 to 0·8 mm. thick. Cable introduced after concreting	117	1·50	1·32–1·68
Metal sheet 0·8 mm. thick. Cable introduced before concreting	31	0·90	0·70–1·10
Metal sheet 0·15 to 0·25 mm. thick. Cable introduced before concreting	449	1·52	1·49–1·55
Metal sheet 0·6 to 0·8 mm. thick. Cable introduced before concreting	568	0·75	0·71–0·79

Mean values of percentage losses per unit length, together with their confidence limits, were also calculated for each type of duct tested, and the results are given in *Table* I.

It should be noted that the values for the losses of prestressing force when metal ducts are used are about 50 per cent. higher than those given by Mr. Cooley, which were based on laboratory tests. It seems advisable, especially for long structures, to determine by tests the losses occurring in each case.

(8) THE 99 PER CENT. CONFIDENCE LIMITS OF THE PRESTRESSING FORCE. —The mean values and confidence limits of the individual losses and errors in assessing the prestressing force given in the foregoing enable maximum and mean percentage errors to be included in the assessment of the prestressing force.

In the case of a cable, 30 ft. long, with a stress of 150,000 lb. per square inch, tensioned from one end, the greatest errors, when the maximum and mean values of the effects causing losses are not allowed for, are 49·1 per cent. and 28 per cent. respectively. If the mean values of the separate effects are allowed for, the greatest error in the assessment of the prestressing force is 8·3 per cent., and the probability of its occurrence is 0·01[8].

In conclusion it can be stated that (1) The losses of prestressing force in

straight cables are regular in character, and present an important problem which needs further investigation. (2) The losses could be limited or prevented entirely. (3) The losses can be taken into account in the design with a high degree of accuracy.

(1) Arne J. Johnson. The Determination of the Design Factor for Reinforced Concrete Structures. Symposium on the Strength of Concrete Structures, London, 1956.

(2) E. H. Cooley. " Friction in Post-tensioned Prestressing Cables." Cement and Concrete Association, London.

(3) Y. Montagnon. Aspects Pratiques de la Precontrainte par cables. " Annales de l'Institut Technique du Batiment et des Travaux Public."

(4) " Symposium on Prestressed Concrete Statically-Indeterminate Structures ", 1954.

(5) W. Zerna. Auslöschen des Spannkraftverlustes infolge Reibung bei Spanngliedern für Spannbeton. " Beton u. Stahlbetonbau ", 1953 and 1955.

(6) J. Zielinski. Badanie strat sily sprezajocej. " Inzynieria i Budownictwo," 1955.

An Unusual Bridge at Moscow.

A TWO-LEVEL reinforced concrete bridge across the river Moskva at Moscow, which is nearing completion, is shown below. The bridge will be the largest in Moscow. The lower deck will be used by trains of the underground system and the upper deck by road vehicles. A path for pedestrians is also provided. A station, with glazed sides and with a capacity of up to 45,000 passengers an hour, will also be included at the lower level; the station will be 46 ft. above the water.

The total length of the bridge is nearly 2650 ft., consisting of a central arch with a span of 650 ft. and approach spans supported on trestles. The approach spans were constructed in the normal way. The central span was built on the bank of the river and then moved by means of winches along two piers, built for the purpose, to two pontoons. It was then floated into position, lifted, and placed on supports which had been previously prepared. The weight of the central span is 5000 tons. The illustration shows the deck and arch of the central span and parts of the adjacent spans, on the pontoons.

Curved Roofs of Large Span.

The roofs of two warehouses completed recently at Paisley for Messrs. Chivas Bros., Ltd., each comprise three 100-ft. spans of corrugated vaults. One building (*Figs.* 1 and 2) is 100 ft. wide and the other is 150 ft. wide. The height to the springing of the vaults is 10 ft. and to the crown about 31 ft.

The design of the structures is in general in accordance with B.S. Code of Practice No. 114 (1957). The roofs, the slabs of which are 2½ in. thick, are designed for a load due to snow of 15 lb. per square foot which is assumed to cover the entire vault or one half only. The pressure of the wind is assumed to be 10 lb. per square foot. A range of temperature of 60 deg. F. was assumed and allowance was made for shrinking of the concrete and for deformation due to spreading of the roof as a result of non-uniform load. Each vault was designed as hinged at the supports and an allowance was then made for partial continuity. The concrete mixture is 1 : 2 : 4 with coarse aggregate of ¾-in. maximum size and coarse sand plus a small amount of fine sand to act as a plasticiser. A compressive strength of about 3800 lb. per square inch was obtained at seven days.

The shape of each vault is an inverted catenary having a rise of about 21 ft. The stresses are therefore mainly compressive, and due to the dead load are about 76 lb. per square inch at the crown and 102 lb. per square inch at the springing. The greatest bending moment due to the combined effect of the dead and imposed loads produces a stress equal to 77 per cent. of the permissible compressive stress. Tensile forces are resisted by the reinforcement. The longitudinal corrugations stiffen the vault against external forces.

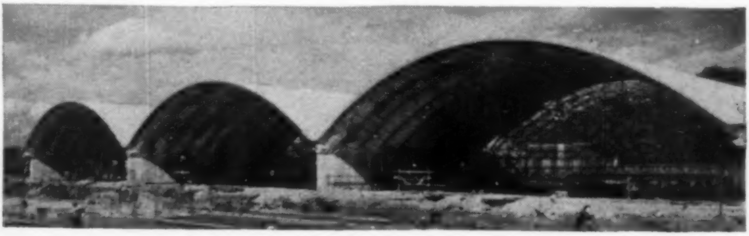


Fig. 1.—Roof During Construction.

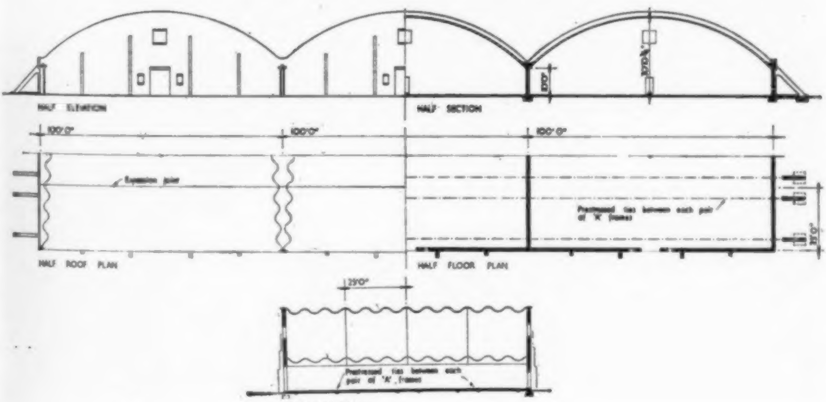


Fig. 2.—Part Cross Sections.

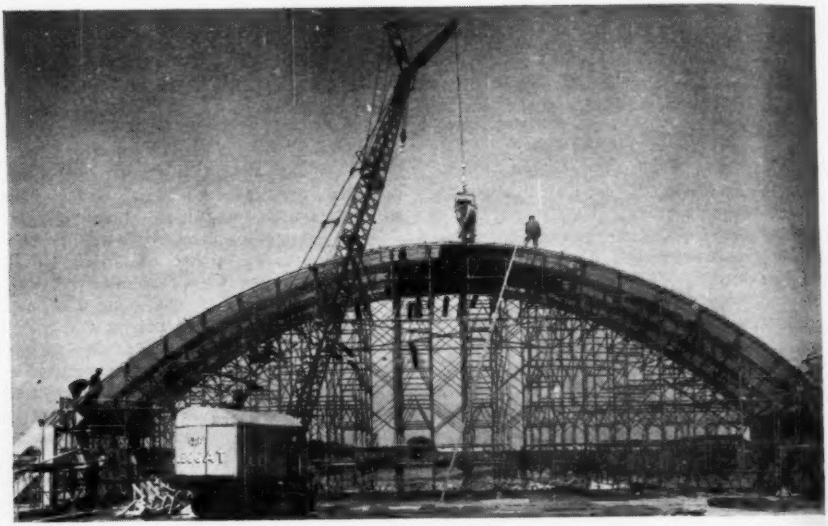


Fig. 3.—View During Construction.

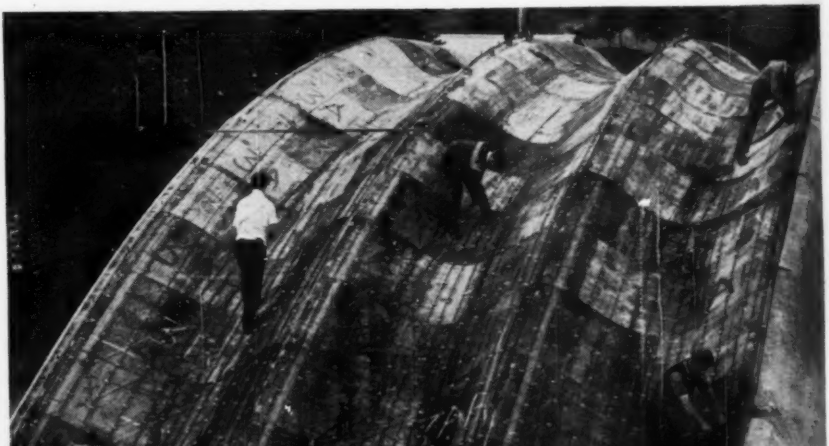


Fig. 4.—Fixing Reinforcement.

The roofs were constructed in sections (*Fig.* 3) 25 ft. wide and of length equal to the three spans. An expansion joint is provided between each section. The centering was erected for one section at a time and comprised steel scaffolding across the uppermost tubes of which small-mesh expanded metal was arranged to form the corrugations. The expanded metal, which weighs 5½ lb. per square yard and was supported at intervals of 8 ft. 4 in. (equal to the pitch of the corrugations), serves as secondary reinforcement. The primary reinforcement (*Fig.* 4) comprises ½-in. and ⅜-in. mild steel bars laid in the valleys and along the crests of the corrugations, and amounts to about 1 lb. per square foot of roof. The cover of concrete over the bars is 1 in.

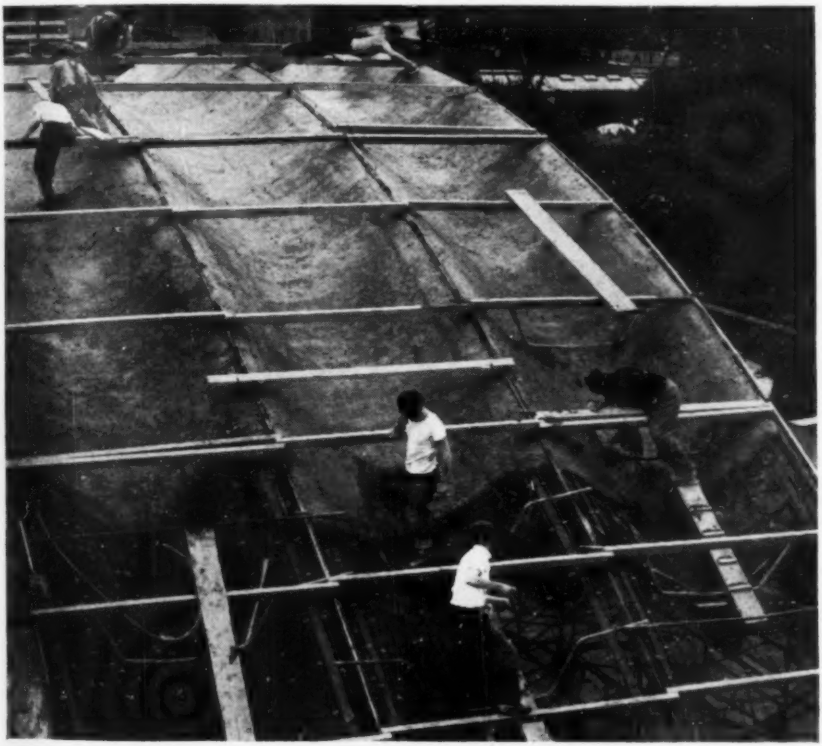


Fig. 5.—Placing the Concrete.

The concrete was placed in a layer 2-in. thick on the expanded metal (*Fig.* 5), and ½ in. of 1 : 3 cement-sand mortar was applied in two coats to the underside. One section 25 ft. wide and 300 ft. long was completed every seven days, that is an average rate of about 1000 sq. ft. daily.

In two similar warehouses now under construction re-usable oil-tempered hardboard or plywood shuttering is used in place of expanded metal, and the secondary reinforcement is mild steel bars. This method obviates rendering on the soffit and facilitates the attachment of an insulating lining to the soffit; such insulation was not required for these buildings nor was it necessary to waterproof the top of the roofs.

The intermediate supports at the

valleys of the roofs are reinforced concrete beams on $13\frac{1}{2}$-in. brick walls which are stiffened by reinforced concrete columns. The walls carry the vertical loads and the columns resist the unbalanced horizontal forces due to non-uniform imposed loads. The outer supports are reinforced concrete beams carried on reinforced concrete A-frames which transmit the thrust from the vaults to the ground. The horizontal component of this thrust is resisted by ties extending between opposite sides of the building. Each tie comprises seven 0·276-in. high-tensile wires 330 ft. long laid in ducts in the floor and tensioned one at a time. The wires pass freely through the bottom horizontal member of the A-frames and are anchored in plates on the outer ends of these members. The walls between the A-frames and the gable walls are of silver-grey sand-lime facing bricks.

The ground floor is a 5-in. reinforced concrete slab on polythene sheeting laid on rolled clinker 3 in. thick.

The architect for the warehouses is Mr. S. Lothian Barclay, and the consulting engineers are Mr. J. H. de W. Waller, D.S.O., O.B.E., and Mr. A. C. Aston. The roof construction is of the form patented by Ctesiphon Construction, Ltd., and described in a paper in the Journal of the Institution of Civil Engineers, Part III, August, 1953. The contractors for the warehouses are Messrs. Hugh Leggat, Ltd.

A Bridge at Lancaster.

A BRIDGE which will carry the Lancaster by-pass road over the River Lune is shown in *Fig.* 1. It will be 44 ft. high and will consist of two open-spandrel arches with spans of 230 ft. Each arch will support a carriageway 26 ft. wide; there will also be a central reserve 37 ft. wide and verges 4 ft. wide.

Two rotary climbing cranes are being used to build the bridge. They are placed at each end between the arches, and are on temporary foundations each of which comprises a block of concrete 12 ft. wide, 14 ft. 9 in. long, and 4 ft. 9 in. deep, supported on five greenheart piles. The jibs of the cranes are about 80 ft. above the level of the water; the cranes are 63 ft. high and the jibs, with an effective radius of 82 ft., can deliver materials to almost any position along the bridge (*Fig.* 2).

The work was designed by the Surveyor's Department of the Lancashire County Council. The general contractors are Sir Lindsay Parkinson & Co., Ltd., and the cranes were supplied by Messrs. Abelson & Co. (Engineers), Ltd.

Fig. 1.

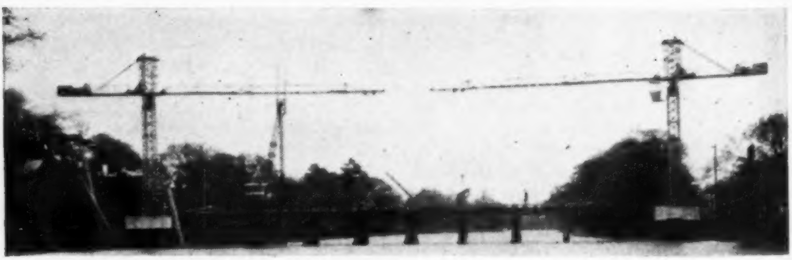


Fig. 2.

Elastic Analysis of Two-dimensional Indeterminate Frames.—II.*

By A. H. DOUGLAS, M.C., M.A., B.A.I., A.M.I.C.E.

Application to Relaxation Method.

The relaxation method is a process for setting up and solving, by tabular methods, arrays of linear simultaneous equations similar to equation (2) which have as variables the final displacements and rotations of the various joints which are free to move. The coefficients are derived from equation (3C) defining U. The

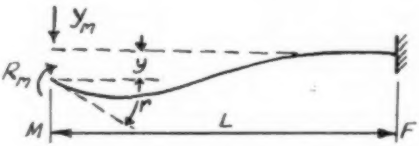


Fig. 3.

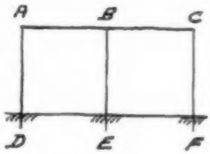


Fig. 4.

equations are not formally stated, but if they are required they can be obtained readily from an operations table.

One or more joints at a time are given movements that tend to transmit the imposed loads progressively from the unsupported joints to the foundations. Because such movements do occur, some energy, however small, is expended to cause them, and therefore the total potential energy of the system is progressively reduced. In the final state of stable equilibrium the potential energy is at a local minimum throughout, and there is therefore no tendency to further movement at any point.

VIII

$$U = \frac{2EI}{L}\left\{\left(r+\frac{y}{L}\right)^2+\left(r+\frac{y}{L}\right)\frac{y}{L}+\left(\frac{y}{L}\right)^2\right\}+C$$

$$= \frac{2EI}{L}\left(r^2+3r.\frac{y}{L}+3\frac{y^2}{L^2}\right)+C$$

since $\theta_M = r$, $\theta_F = 0$, and $\varphi = -\frac{y}{L}$ i.e. opposite sign to θ_M

Differentiating, and neglecting signs for the moment

$$\frac{dU}{dy} = \frac{2EI}{L}\left(3\frac{r}{L}+6\frac{y}{L^2}\right) = y_M$$

$$\frac{dU}{dr} = \frac{2EI}{L}\left(2r+3\frac{y}{L}\right) = R_M$$

Differentiating again:

$$\frac{d^2U}{dy^2} = \frac{12EI}{L^3};\quad \frac{d^2U}{dy\,dr} = \frac{d^2U}{dr\,dy} = \frac{6EI}{L^2};\quad \frac{d^2U}{dr^2} = \frac{4EI}{L}$$

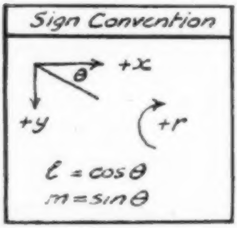
* Continued from April, 1959, in which equations (1) to (8) and I to VII and Figs. 1 and 2 are given.

Determination of Basic Influence Coefficients.—Let MF (*Fig.* 3) be an element of length L of a typical frame. The element is free to move at M and is fixed at F and is subjected to a force Y_M and a moment R_M. Starting with equation (3C) and differentiating twice, equations VIII are obtained. These second differential coefficients represent the actions at M due to unit deformations at M. Similarly, from the overall conditions of equilibrium for the member, the corresponding actions at F due to unit deformation at M are

$$\frac{12EI}{L^3}, \quad \frac{6EI}{L^2}, \quad \text{and} \quad \frac{6EI}{L} - \frac{4EI}{L} = \frac{2EI}{L}.$$

These coefficients are analogous to the corresponding coefficients of the moment-area method, but in this case they have to be adapted to the sign convention of the system of co-ordinates used. In this method the reactions of the members on their constraints are considered, and not the actions of external forces on the

$$\text{IX} \left| \begin{aligned}
-xX_m = +xX_F &= \frac{EA}{L}l^2 + 12\frac{EI}{L^3}m^2 \\
\left.\begin{aligned} -xY_m &= +xY_F \\ = -yX_m &= +yX_F \end{aligned}\right\} &= \frac{EA}{L}lm - 12\frac{EI}{L^3}lm \\
-yY_m = +yY_F &= \frac{EA}{L}m^2 + 12\frac{EI}{L^3}l^2 \\
\left.\begin{aligned} -xR_m &= -xR_F \\ = -rX_m &= +rX_F \end{aligned}\right\} &= -6\frac{EI}{L^2}m \\
\left.\begin{aligned} -yR_m &= -yR_F \\ = -rY_m &= +rY_F \end{aligned}\right\} &= +6\frac{EI}{L^2}l \\
rR_m &= -4\frac{EI}{L} \\
rR_F &= -2\frac{EI}{L}
\end{aligned}\right.$$



where $-xX_m$ = X reaction at M due to unit deformation in x direction at M, etc.

members. In its complete form the method also takes into account the longitudinal strains and stresses. When all these adjustments have been made the complete set of eighteen basic influence coefficients for two-dimensional analysis given in equations IX is obtained. Deformations can occur only at M, but they cause reactions at M and F.

The last two coefficients in IX are the same as the moment-distribution coefficients of Professor Hardy Cross's method, except that his method is concerned with actions and not reactions, and the fixing moments are applied to the members and not to the joints. Using equation 3A instead of 3C it can be shown that, for a member which is pin-jointed at either end, the numerators of these coefficients (if they do not vanish) reduce to a common value of $3EI$.

Example.—For a doubly-fixed rectangular frame (*Fig.* 4), assume $L = 10$ ft. and $EI = 10{,}000$ throughout the rectangular frame. If longitudinal strains are neglected, then x_A, x_B, and x_C are equal. Vertical strains, that is all coefficients

involving y, are also neglected. Then $\frac{2EI}{L} = 2000$, $\frac{6EI}{L^2} = 600$, and $\frac{12EI}{L^3} = 120$, and the coefficients in *Tables* A and B can be obtained.

Normally, a relaxation table follows in which the values from the operations table are used to solve a particular problem by relaxation methods. In this example, as there are only four independent variables, a general solution will be obtained by matrix methods, and then applied in a relaxation table to find the remaining reactions and to check the previous work. Each relevant column of *Table* B gives the coefficients for one version of the minimum-energy equation (2) as shown in equations X (page 178), for which the general solution is shown in

TABLE A.—BASIC INFLUENCE COEFFICIENTS.

	A		B			C	
	AD	AB	BA	BE	BC	CB	CF
l / m	0 / +1	+1 / 0	−1 / 0	0 / +1	+1 / 0	−1 / 0	0 / +1
$-xX_M = +xX_F$	+120, $\underline{+120}$	—	—	+120, $\underline{+120}$	—	$\underline{+120}$	+120
$-xR_M = -xR_F$ = $-rX_M = +rX_F$	−600, $\underline{-600}$	0	0	−600, $\underline{-600}$	0	0, $\underline{-600}$	−600
rR_M	−4000, $\underline{-8000}$	−4000	−4000	−4000, $\underline{-12000}$	−4000	−4000, $\underline{-8000}$	−4000
rR_F	−2000	−2000	−2000	−2000	−2000	−2000	−2000

Note: Figures underlined are suffix M coefficients; remainder are suffix F.

TABLE B.—OPERATIONAL INFLUENCE COEFFICIENTS.
(OPERATIONS TABLE.)

		D		A		B		C		E		F	
		x	R	x	R	x	R	x	R	x	R	x	R
A	x	+120	+600	−120	+600	—	0						
	r	−600	−2000	+600	−8000	0	−2000						
B	x			—	0	−120	+600	—	0	+120	+600		
	r			0	−2000	+600	−12000	0	−2000	−600	−2000		
C	x					—	0	−120	+600			+120	+600
	r					0	−2000	+600	−8000			−600	−2000

Note: The semi-graphical nature of this Table makes it easy to check the accuracy of the various postings; e.g. $xX_{AD} = xX_F = +120$; $xX_{AA} = xX_M = -120$; $rX_{BC} = rX_F = 0$; $rR_{BB} = rR_M = -12000$; *etc.*

$$\begin{vmatrix} x \\ r_A \\ r_B \\ r_C \end{vmatrix} = \frac{1}{184300} \begin{vmatrix} 704\cdot0 & 48\cdot0 & 19\cdot2 & 48\cdot0 \\ 48\cdot0 & 27\cdot4 & -2\cdot9 & 4\cdot3 \\ 19\cdot2 & -2\cdot9 & 17\cdot3 & -2\cdot9 \\ 48\cdot0 & 4\cdot3 & -2\cdot9 & 27\cdot4 \end{vmatrix} \times \begin{vmatrix} \Sigma X \\ R_A \\ R_B \\ R_C \end{vmatrix} \quad \cdots (9)$$

$$
\text{X}\quad
\begin{array}{lllll}
-120x_A & +600r_A & & & = -X_A \\
+600x_A & -8000r_A & -2000r_B & & = -R_A \\
-120x_B & & +600r_B & & = -X_B \\
+600x_B & -2000r_A & -12000r_B & -2000r_C & = -R_B \\
-120x_C & & & +600r_C & = -X_C \\
+600x_C & & -2000r_B & -8000r_C & = -R_C
\end{array}
$$

As $x_A \equiv x_B \equiv x_C$, this reduces to:

$$
\begin{vmatrix} -360 & +600 & +600 & +600 \\ +600 & -8000 & -2000 & 0 \\ +600 & -2000 & -12000 & -2000 \\ +600 & 0 & -2000 & -8000 \end{vmatrix} \times \begin{vmatrix} x \\ r_A \\ r_B \\ r_C \end{vmatrix} = \begin{vmatrix} -\Sigma X \\ -R_A \\ -R_B \\ -R_C \end{vmatrix}
$$

$$
\text{XI}\quad
\begin{aligned}
x &= \frac{704{\cdot}0}{184300} = +0{\cdot}00382\,\Sigma X \\
r_A &= \frac{48{\cdot}0}{184300} = +0{\cdot}00026\,\Sigma X \\
r_B &= \frac{19{\cdot}2}{184300} = +0{\cdot}00010\,\Sigma X \\
r_C &= \frac{48{\cdot}0}{184300} = +0{\cdot}00026\,\Sigma X
\end{aligned}
$$

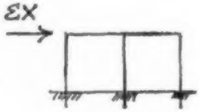


TABLE C.—RELAXATION TABLE FOR FIXED DOUBLE RECTANGULAR FRAME.

	D		A		B		C		E		F	
	X	R	X	R	X	R	X	R	X	R	X	R
Load					+10·0	=ΣX						
$+0{\cdot}0382x_A$	+4·58	+22·9	−4·58	+22·9	—	0						
$+0{\cdot}0026r_A$	−1·56	−5·2	+1·56	−20·8	0	−5·2						
$+0{\cdot}0382x_B$			—	0	−4·58	+22·9	—	0	+4·58	+22·9		
$+0{\cdot}0010r_B$			0	−2·0	+0·60	−12·0	0	−2·0	−0·60	−2·0		
$+0{\cdot}0382x_C$					—	0	−4·58	+22·9			+4·58	+22·9
$+0{\cdot}0026r_C$					0	−5·2	+1·56	−20·8			−1·56	−5·2
Residuals	+3·02	+17·7	−3·02	+0·1	−3·98 −10·0	+0·5	−3·02	+0·1	+3·98	+20·9	+3·02	+17·7
X_D	+0·30ΣX											
X_E									+0·40ΣX			
X_F											+0·30ΣX	
R_D		+0·177ΣX.L										
R_E										+0·209ΣX.L		
R_F												+0·177ΣX.L

TABLE D.—ADDENDUM TO OPERATIONS TABLE (TABLE B).

	D		A		B		C		E		F	
	X	R	X	R	X	R	X	R	X	R	X	R
$a = \Sigma x$	+120	+600	−120	+600	−120	+600	−120	+600	+120	+600	+120	+600
$b = r_A + r_C$	−600	−2000	+600	−8000	0	−4000	+600	−8000			−600	−2000
$c = 0{\cdot}4 r_B$				−800	+240	−4800		−800	−240	−800		
$d = 14{\cdot}7a$	+1760	+8800	−1760	+8800	−1760	+8800	−1760	+8800	+1760	+8800	+1760	+8800
$e = b + c + d$	+1160	+6800	−1160	0	−1520	0	−1160	0	+1520	+8000	+1160	+6800
					−3840							
$f = \frac{e}{3840}$	+0·30	+1·77	−0·30	0	−0·40	0	−0·30	0	+0·40	+2·08	+0·30	+1·77

equation (9), which is the deformation analogue of equation (8) in Part I. For simple side sway, that is, $R_A = R_B = R_C = 0$, equation (9) gives the results in XI (page 178). Inserting these values in a relaxation table and assuming that $\Sigma X = 10$ tons, the results given in *Table* C are obtained.

An experienced designer could obtain quickly the same result for this particular loading by group relaxation derived from the operations table as given in *Table* D, which is an addendum to *Table* B. However, the general solution in equation (9) has the advantage of being valid for all arrangements of loads.

Although direct extrapolation from existing solutions is not so easy with the relaxation method as with the moment-area method, existing results can often be applied to the analysis of more complex frames of the same type. For example, the addition of a diagonal brace between D and B in the previous example will not alter appreciably the relative values of the various deformations under side sway only, but it will reduce considerably their actual amounts.

In a frame designed to carry heavy loads the strains due to direct tension and compression are comparable with those due to bending, and cannot be neglected, particularly when the frame is part of a three-dimensional structure. A braced framework, of the type referred to already, was analysed and the effect of grouting on the stresses and strains in the bolts, which secure the column AD against uplift, was determined. There are ten unknowns in this problem: the vertical movement (if any) of the left-hand column base, and the *x*, *y*, and *r* deformations at A, B, and C.

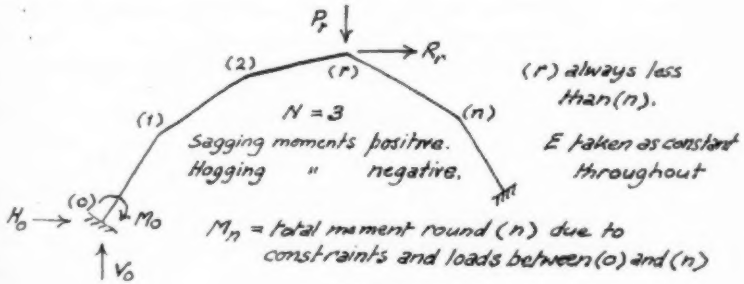


Fig. 5.

$$6E.U = \Sigma \frac{1}{K_n}(M_{n-1}^2 + M_{n-1}M_n + M_n^2) + C$$

Taking moments round (n):

$$M_n = V_0 x_n + H_0 y_n + M_0 + M$$

where M = moment round joint (n) of that part of the external load operating between (0) and (n) *It follows that:*

$$\frac{dM_n}{dV_0} = x_n \; ; \; \frac{dM_n}{dH_0} = y_n \; ; \; \frac{dM_n}{dM_0} = 1$$

Expanding the equation for U:

$$6E.U = \cdots + \frac{1}{K_n}(M_{n-1}^2 + M_{n-1}M_n + M_n^2)$$

XII

$$+ \frac{1}{K_{n+1}}(M_n^2 + M_n M_{n+1} + M_{n+1}^2) + \cdots$$

Differentiating (and neglecting the multiplier 6E, which drops out later):

$$\frac{dU}{dV_0} = \cdots + \frac{dU}{dM_{n-1}} \cdot \frac{dM_{n-1}}{dV_0} + \frac{dU}{dM_n} \cdot \frac{dM_n}{dV_0} + \frac{dU}{dM_{n+1}} \cdot \frac{dM_{n+1}}{dV_0} + \cdots$$

$$= \Sigma M_n \left\{ \frac{1}{K_n} x_{n-1} + 2\left(\frac{1}{K_n} + \frac{1}{K_{n+1}}\right) x_n + \frac{1}{K_{n+1}} x_{n+1} \right\}$$

$$= \Sigma M_n . \lambda_n = 0 \text{ (since there can be no deformation at this point)}$$

$$= V_0 \Sigma x_n \lambda_n + H_0 \Sigma y_n \lambda_n + M_0 \Sigma \lambda_n + \Sigma M . \lambda_n = 0$$

Differentiating again:

$$\frac{d^2U}{dV_0^2} = \Sigma x_n \lambda_n \; ; \; \frac{d^2U}{dV_0 dH_0} = \Sigma y_n \lambda_n \; ; \; \frac{d^2U}{dV_0 dM_0} = \Sigma \lambda_n$$

Similarly:

$$\frac{dU}{dH_0} = V_0 \Sigma x_n \mu_n + H_0 \Sigma y_n \mu_n + M_0 \Sigma \mu_n + \Sigma M \mu_n = 0$$

$$\frac{d^2U}{dH_0^2} = \Sigma y_n \mu_n \; ; \; \frac{d^2U}{dH_0 dV_0} = \Sigma x_n \mu_n \; ; \; \frac{d^2U}{dH_0 dM_0} = \Sigma \mu_n$$

and

$$\frac{dU}{dM_0} = V_0 \Sigma x_n \nu_n + H_0 \Sigma y_n \mu_n + M_0 \Sigma \nu_n + \Sigma M \nu_n = 0$$

$$\frac{d^2U}{dM_0^2} = \Sigma \nu_n \; ; \; \frac{d^2U}{dM_0 dV_0} = \Sigma x_n \nu_n \; ; \; \frac{d^2U}{dM_0 dH_0} = \Sigma y_n \nu_n$$

where λ_n, μ_n and ν_n are basic influence coefficients with the following values:

$$\left.\begin{aligned} \lambda_n &= \tfrac{2}{3} x_n \nu_n + \frac{1}{K_n} x_{n-1} + \frac{1}{K_{n+1}} x_{n+1} \\ \mu_n &= \tfrac{2}{3} y_n \nu_n + \frac{1}{K_n} y_{n-1} + \frac{1}{K_{n+1}} y_{n+1} \\ \nu_n &= 3\left(\frac{1}{K_n} + \frac{1}{K_{n+1}}\right) \end{aligned}\right\} \quad \ldots\ldots (10)$$

(continued)

The corresponding operational influence coefficients are:

$$\left.\begin{array}{l} \Sigma x_n\lambda_n ;\ \Sigma y_n\mu_n ;\ \Sigma \nu_n ; \\ \Sigma y_n\lambda_n = \Sigma x_n\mu_n ; \\ \Sigma\lambda_n = \Sigma x_n\nu_n ; \\ \Sigma\mu_n = \Sigma y_n\nu_n ; \end{array}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

The equations expressed in matrix form are:

$$\begin{vmatrix} \Sigma x_n\lambda_n & \Sigma y_n\lambda_n & \Sigma\lambda_n \\ \Sigma y_n\lambda_n & \Sigma y_n\mu_n & \Sigma\mu_n \\ \Sigma\lambda_n & \Sigma\mu_n & \Sigma\nu_n \end{vmatrix} \times \begin{vmatrix} V_o \\ H_o \\ M_o \end{vmatrix} = \begin{vmatrix} -\Sigma M\lambda_n \\ -\Sigma M\mu_n \\ -\Sigma M\nu_n \end{vmatrix}$$

for which the general solution is:

XII contd.

$$\begin{vmatrix} V_o \\ H_o \\ M_o \end{vmatrix} = \frac{1}{\Delta} \begin{vmatrix} a_{11} & a_{12} & a_{13} \\ a_{21} & a_{22} & a_{23} \\ a_{31} & a_{32} & a_{33} \end{vmatrix} \times \begin{vmatrix} -\Sigma M\lambda_n \\ -\Sigma M\mu_n \\ -\Sigma M\nu_n \end{vmatrix} \quad \ldots\ldots\ldots (12)$$

where:

$$a_{11} = \Sigma y_n\mu_n . \Sigma\nu_n - (\Sigma\mu_n)^2$$

$$a_{12} = -\Sigma y_n\lambda_n . \Sigma\nu_n + \Sigma\mu_n . \Sigma\lambda_n = a_{21}$$

$$a_{13} = \Sigma y_n\lambda_n . \Sigma\mu_n - \Sigma y_n\mu_n . \Sigma\lambda_n = a_{31}$$

$$a_{22} = \Sigma x_n\lambda_n . \Sigma\nu_n - (\Sigma\lambda_n)^2$$

$$a_{23} = -\Sigma x_n\lambda_n . \Sigma\mu_n + \Sigma y_n\lambda_n . \Sigma\lambda_n = a_{32}$$

$$a_{33} = \Sigma x_n\lambda_n . \Sigma y_n\mu_n - (\Sigma y_n\lambda_n)^2$$

$$\Delta = \Sigma x_n\lambda_n . a_{11} + \Sigma y_n\lambda_n . a_{12} + \Sigma\lambda_n . a_{13}$$

For a particular arrangement of loads, approximate solutions for the two extremes of case (i) no grout and case (ii) complete fixity at ground level can be obtained (on about one page of foolscap in each case) by relaxation methods. A general solution in the form of a 10 × 10 reciprocal matrix was obtained in about one hour, using a digital computing machine. Most of this time was used to punch cards to fit an existing programme, which was not designed for structural problems. The computation occupied only a few minutes. Most of the results obtained by relaxation methods were within 5 per cent. of the values given by this accurate solution, and were therefore sufficiently accurate for the purpose. For the assumed arrangement of loads the stresses in the holding-down bolts, when fully secured by grout, were about 20 per cent. greater, and the deformations at A about 60 per cent. less, than the stresses if the bolts were free to stretch along their whole length.

The main advantages of a general solution (now obtainable at reasonable cost

for problems containing up to thirty unknowns) are that it is valid for all arrangements of load, that it requires simple arithmetic only, and that systematical comparison of its components, by rows and columns, gives a clearer understanding of the general behaviour of the frame.

Application to Strain-energy Method.

The following solution is valid for all fixed polygonal arches (*Fig.* 5), regardless of the number of members or their precise arrangement, and can be adapted easily to cases in which one or both ends are hinged. A similar solution for a

TABLE E.—EVALUATION OF BASIC AND OPERATIONAL INFLUENCE COEFFICIENTS FOR FIXED GABLE FRAMES.

	0	1	2	3	4	Σ
$\frac{1}{K_n}$	0	1	1·5	1·5	1	
$\frac{1}{K_{n+1}}$	1	1·5	1·5	1	0	
v_n	3	7·5	9	7·5	3	30
x_{n-1}	0	0	0	$\frac{1}{2}l$	l	
x_n	0	0	$\frac{1}{2}l$	l	l	
x_{n+1}	0	$\frac{1}{2}l$	$\frac{1}{2}l$	l	0	
y_{n-1}	0	0	h	$1{\cdot}3h$	h	
y_n	0	h	$1{\cdot}3h$	h	0	
y_{n+1}	h	$1{\cdot}3h$	h	0	0	
$\frac{1}{K_n}x_{n-1}$	0	0	0	$0{\cdot}75l$	l	
$\frac{1}{K_{n+1}}x_{n+1}$	0	$0{\cdot}75l$	$1{\cdot}5l$	l	0	
$\frac{1}{K_n}y_{n-1}$	0	0	$1{\cdot}5h$	$1{\cdot}95h$	h	
$\frac{1}{K_{n+1}}y_{n+1}$	h	$1{\cdot}95h$	$1{\cdot}5h$	0	0	
$x_n v_n$	0	0	$4{\cdot}5l$	$7{\cdot}5l$	$3l$	$15l = \Sigma\lambda_n$
$y_n v_n$	0	$7{\cdot}5h$	$11{\cdot}7h$	$7{\cdot}5h$	0	$26{\cdot}7h = \Sigma\mu_n$
$\frac{2}{3}x_n v_n$	0	0	$3{\cdot}0l$	$5{\cdot}0l$	$2l$	
$\frac{2}{3}y_n v_n$	0	$5{\cdot}0h$	$7{\cdot}8h$	$5{\cdot}0h$	0	
λ_n	0	$0{\cdot}75l$	$4{\cdot}5l$	$6{\cdot}75l$	$3l$	$15l = \Sigma x_n v_n$
μ_n	h	$6{\cdot}95h$	$10{\cdot}8h$	$6{\cdot}95h$	h	$26{\cdot}7h = \Sigma y_n v_n$
$x_n\lambda_n$	0	0	$2{\cdot}25l^2$	$6{\cdot}75l^2$	$3l^2$	$12l^2$
$y_n\lambda_n$	0	$0{\cdot}75hl$	$5{\cdot}85hl$	$6{\cdot}75hl$	0	$13{\cdot}35hl = \Sigma x_n\mu_n$
$x_n\mu_n$	0	0	$5{\cdot}4hl$	$6{\cdot}95hl$	hl	$13{\cdot}35hl = \Sigma y_n\lambda_n$
$y_n\mu_n$	0	$6{\cdot}95h^2$	$14{\cdot}04h^2$	$6{\cdot}95h^2$	0	$27{\cdot}94h^2$

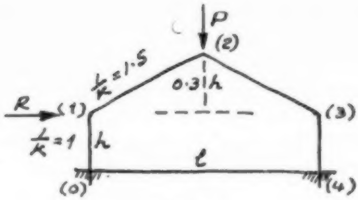


Fig. 6.

hinged arch was published by Dr. E. H. Bateman.[1] Tabular methods can be used to find the influence coefficients. As the degree of redundancy is three, a general solution can be produced easily for any particular family of arches, without considering the arrangement of loads to be applied. The arrangement of

$$a_{11} = 27\cdot94h^2 \times 30 - 26\cdot7h \times 26\cdot7h \qquad = 125\cdot25h^2$$
$$a_{12} = -13\cdot35hl \times 30 + 26\cdot7h \times 15l \qquad = 0$$
$$a_{13} = 13\cdot35hl \times 26\cdot7h - 27\cdot9h^2 \times 15l \qquad = -62\cdot62h^2l$$
$$a_{22} = 12l^2 \times 30 - 15l \times 15l \qquad = 135\cdot0l^2$$
$$a_{23} = -12l^2 \times 26\cdot7h + 13\cdot35hl \times 15l \qquad = -120\cdot2hl^2$$
$$a_{33} = 12l^2 + 27\cdot94h^2 - 13\cdot35hl \times 13\cdot35hl \qquad = 157\cdot1h^2l^2$$
$$\Delta = 12l^2 \times 125\cdot25h^2 + 15l \times -62\cdot62h^2l \qquad = 563\cdot5h^2l^2$$

The general solution for this family of frames is, therefore,

$$\begin{vmatrix} V_0 \\ H_0 \\ M_0 \end{vmatrix} = \frac{1}{563\cdot5} \begin{vmatrix} \frac{125\cdot25}{l^2} & 0 & \frac{-62\cdot62}{l} \\ 0 & \frac{135\cdot00}{h^2} & \frac{-120\cdot20}{h} \\ \frac{-62\cdot62}{l} & \frac{-120\cdot20}{h} & 157\cdot10 \end{vmatrix} \times \begin{vmatrix} -\Sigma M\lambda_n \\ -\Sigma M\mu_n \\ -\Sigma M\nu_n \end{vmatrix} \qquad (13)$$

XIII *It now remains to develop a suitable expression for M:*

(i) for loads at joint r:

$$M = \Sigma_1^n P_r(x_n \sim x_r) + \Sigma_1^n R_r(y_n \sim y_r)$$

(ii) for mid-span loading between r and $r+1$, equation 3B gives

$$6E.U = \cdots + \frac{1}{K_r}\left(M_{r-1}^2 + M_{r-1}M_r + M_r^2\right) + \frac{1}{K_{r+1}}\left\{(M_r - F_r)^2 + (M_r - F_r)(M_{r+1} - F_{r+1}) + (M_{r+1} - F_{r+1})^2\right\}$$

from which it follows that:

$$\frac{dU}{dV_0} = \Sigma M_n\lambda_n - \frac{1}{K_{r+1}}\left\{(2F_r + F_{r+1})x_r + (F_r + 2F_{r+1})x_{r+1}\right\} = 0$$

Thus the complete expression for $M\lambda_n$ is:

$$M\lambda_n = \Sigma_1^n P_r(x_n \sim x_r)\lambda_n + \Sigma_1^n R_r(y_n \sim y_r)\lambda_n - \Sigma_1^n \frac{1}{K_{r+1}}(2F_r + F_{r+1})x_r - \Sigma_1^n \frac{1}{K_{r+1}}(F_r + 2F_{r+1})x_{r+1} \quad \ldots\ldots(14)$$

and similarly for $M\mu_n$ and $M\nu_n$ (with corresponding substitutions for x_r and x_{r+1} in the second part of the expression).

loads is expressed in the form of coefficients by the same tabular methods used for the general solution.

Assume that loads are applied at the joints only (loads applied at mid-span are considered later). Starting from equation (3B) and the minimum energy equation (1), since the structure is linear, equations XII are obtained.

The tabular method is illustrated by the application of the general solution to a gable frame with the columns having fixed ends (*Fig.* 6) and is given in *Table* E and equations XIII and XIV. In practice the method would not be used for such a simple case, as it has advantages only when it is used for complex

XIV

Assume a single horizontal load R at joint 1 as in Fig. 6.
Equation 14 in association with Table E gives

	0	1	2	3	4	Σ
$-M = -R(y_n \sim y_1)$	0	0	$+0{\cdot}3hR$	0	$-hR$	
$-M\lambda_n$	0	0	$+1{\cdot}35hlR$	0	$-3hlR$	$-1{\cdot}65hlR$
$-M\mu_n$	0	0	$+3{\cdot}24h^2R$	0	$-h^2R$	$+2{\cdot}24h^2R$
$-M\nu_n$	0	0	$+2{\cdot}7hR$	0	$-3hR$	$-0{\cdot}30hR$

From equation 13 :

$$563{\cdot}5\,V_0 = (125{\cdot}25 \times -1{\cdot}65 - 62{\cdot}62 \times -0{\cdot}30)\frac{h}{l}P \quad = -187{\cdot}7\frac{h}{l}R$$

$$\therefore\ V_0 = -0{\cdot}33\frac{h}{l}R$$

$$563{\cdot}5\,H_0 = (135{\cdot}0 \times +2{\cdot}24 - 120{\cdot}2 \times -0{\cdot}30)P \quad = +338{\cdot}3R$$

$$\therefore\ H_0 = +0{\cdot}60R$$

$$563{\cdot}5\,M_0 = (-62{\cdot}62 \times -1{\cdot}65 - 120{\cdot}20 \times +2{\cdot}24 + 157{\cdot}10 \times -0{\cdot}30)Rh \quad = -212{\cdot}9Rh$$

$$\therefore\ M_0 = -0{\cdot}378Rh$$

Note that these are external forces acting on the framework; also that the quantities in the last column are (when divided by 6E) the deformations which R would cause at joint 0 if that joint were free to more.

Similarly, for a single vertical load P at joint 2 (Fig 6)

$$V_0 = +0{\cdot}50P;\quad H_0 = -0{\cdot}\ 1\frac{l}{h}P;\quad M_0 = +0{\cdot}076Pl$$

TABLE F.

		Single	2-bay	3-bay
Outer column bases	H	0·50R	0·30R	0·22R
	V	0·43R	0·22R	0·17R
	M	0·29R	0·18R	0·13R
Inner column bases	H		0·40R	0·28R
	V		Zero	0·07R
	M		0·21R	0·15R

arches with a complex arrangement of loads, or in investigating the effects of changes of shape, strength, stiffness, and arrangement of loads.

Conclusions.—With the computing services now available, particular and general solutions (in matrix form) for common types of frames could be made available to structural engineers. From these results, modified where necessary by considerations of ultimate strength and stability,[2, 3] charts could be prepared for rapid design, as has been done successfully in the case of multiple-story buildings.[4]

In the case of a rectangular frame with fixed columns subjected to simple side sway (like the square two-bay frame in *Fig.* 4), the forces and moments on the column bases for frames with one, two, and three bays are found to be as in *Table* F. This information is sufficient to indicate the probable behaviour of a four-bay rectangular frame of similar proportions subjected to a similar force. If similar figures were available for other ratios of h to l and I_h to I_l, and for other types of loading, they would form a reliable guide for the rapid design of this type of frame.

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(2) Baker, A. L. L. Recent Research in Reinforced Concrete and its Application to Design. Journ. I.C.E., Feb. 1951 and Aug. 1953.
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Vibration Caused by Driving Piles.

In a paper entitled "Vibration Control in Piling and Blasting" read by Mr. J. H. A. Crockett before the Reinforced Concrete Association, the author gave some examples of vibrations from these causes that had resulted in unusual damage to existing structures or structures in course of erection.

In the course of his remarks the author stated that when driving commences a pile is in an unstable condition and oscillates freely, thereby causing much of the excess of energy to be transmitted as horizontal vibrations in the comparatively soft ground near the surface. These waves, which may be of large amplitude and low frequency, may affect adversely adjacent shallow foundations. As driving progresses a pile becomes steadier laterally and the excess energy is transmitted more directly to the ground in the form of vertical impulses the shock of which can be transmitted to adjacent buildings. As final set is approached the ground vibrates less, the excess energy being transmitted to the harder and denser stratum at the toe of the pile; the effect of these vibrations does not appear so much at the surface, and this stage is probably less dangerous to adjacent buildings than intermediate stages of driving. Pile-driving or other source of vibration adjacent to a reinforced concrete building in course of construction may result in complete failure of bond between the reinforcement and the concrete.

Mr. F. E. Wentworth-Sheilds.

An honorary degree of Doctor of Science is to be conferred on Mr. F. E. Wentworth-Sheilds, O.B.E., by Southampton University on July 3. Mr. Wentworth-Sheilds, who is now in his 90th year, became an Associate Member of the Institution of Civil Engineers in 1895, a member in 1905, and was elected President in 1944.

A Guide Astray.

A REVISION is available of a publication* prepared by the Committee of Co-operative Research of the Road Research Laboratory of the Department of Scientific and Industrial Research with the Cement & Concrete Association and first published in 1949. Its purpose is to present information on the latest practice in the design and construction of concrete roads, and particularly on methods that have been introduced during the past ten years. Useful features are recommended thicknesses of bases and of concrete slabs, amounts of reinforcement, and the spacing and width of joints that are now generally accepted as good practice. The methods of cutting joints after the concrete is placed which have been introduced from the Continent during the past few years are briefly described. It is stated that there is still no reliable formulæ for use in the design of concrete roads that will be suitable in all cases, but information is given on designs that have proved satisfactory on different types of soil and for carrying different volumes of traffic.

The Guide could have been much improved if more care had been taken in its preparation, if the method of giving information in the form of questions and answers had been abandoned in favour of simple sub-headings, and if fewer words had been used. For example, more than a hundred words are used to inform a highway engineer that a road should be able to carry traffic for a long period with little maintenance. A paragraph of six lines has a heading of three lines in the form of a question—in a more rational presentation the heading would have comprised two words only. In other cases headings of more than a dozen words in the form of a question could with advantage be replaced by one word.

Among the useless information offered are the following: "The more variable the strength of the concrete the greater will be the difference between the average strength and the minimum." "The thickness of loose layer to be used and the number of passes required depend on the type of machine used to compact the soil and the soil type." "The output per man hour is increased by the use of power-propelled vibrating machines so that if the plant is carefully chosen the cost of production may be reduced and the speed increased." "A road surface should afford safe and comfortable travel." These are indeed profound truths, but they are more suitable in a kindergarten than in a guide for highway engineers.

We are told that "well-built concrete roads do not suffer from weathering even where the traffic is only occasional and light", from which it can be inferred that the more the traffic the less the weathering. It is stated that "For works involving between one-half and three miles of two-lane carriageway where the compressive strength specified is less than 4000 lb. per square inch at 28 days the standard of control should be dependent on the size and importance of the work", but that if the work extends to less than half a mile "a less rigid standard of control is appropriate". It is doubtful if many highway engineers or road users will agree that an inferior road is satisfactory merely because it is not quite half a mile in length.

The use of the word pavement to describe a road or carriageway in this country is not only wrong but dangerous. Because so much of our knowledge of concrete roads and methods of constructing them is copied from the U.S.A. and the Continent, it is perhaps natural that the reading of foreign literature will lead to the use of foreign terms. But in this country the word "pavement" is derived from the days when only the sides of urban streets were paved for the convenience of pedestrians, and here the word pavement still means a footpath beside a carriageway. Large sums of money are being spent to encourage pedestrians to keep to the pavement and not to step on to the road without making sure that it is safe to do so. Posters tell us that there is Death on the Road. According to the unthinking compilers of this book a modern motorway is all pavement and no road, and presumably we should now be warned that there is Death on the Pavement. It is disappointing to find such slipshod work sponsored by the Department of Scientific and Industrial Research and bearing the imprint "Crown Copyright Reserved".

* "GUIDE TO CONCRETE ROAD CONSTRUCTION." 74 pages. (London: Cement and Concrete Association. Gratis.)

Production of Prestressed Bridge Beams.

To the left of *Fig.* 1 are shown some hog-back prestressed concrete beams for three road bridges over the railway at Bishops Stortford being made on a prestressing bed 235 ft. long. The beams are up to 69 ft. 6 in. long and are of rectangular cross section with a small projection along each bottom edge to support a reinforced concrete slab. The slab, which is cast in place, acts in combination with the beams and comprises the structural deck of each bridge.

The prestressing force in 0·276-in. wires in the largest beams is about 500 tons. In order to reduce the tendency of the prestressing force to produce tensile stresses near the ends of the beams, in some of the beams the wires are encased in P.V.C. tubes of 9 mm. diameter for lengths of 5 ft. to 15 ft. from the ends to prevent bond between the wires and the concrete.

The prestressing bed is of unusual construction. It comprises three parallel walls extending the full length, and between each pair of walls there are two prestressing lines. All three walls are used to resist the pull in the tensioned wires. Each of the prestressing beds can be used separately for a tensioning force of 250 tons, as on the right of *Fig.* 1, or two beds can be combined for a tensioning force of 500 tons as shown to the left of *Fig.* 1.

The procedure is to place the wires in position, anchor them in a steel cross-head at one end, and apply an initial tension of about 400 lb. in each wire. The wires are then fully tensioned in pairs and anchored at the tensioning end,

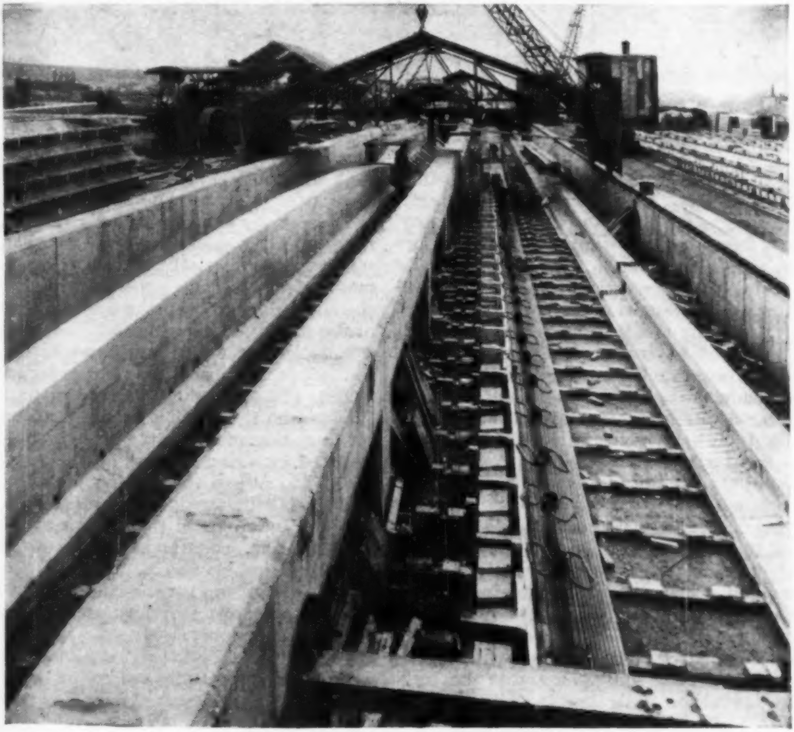


Fig. 1.—The Prestressing Beds.

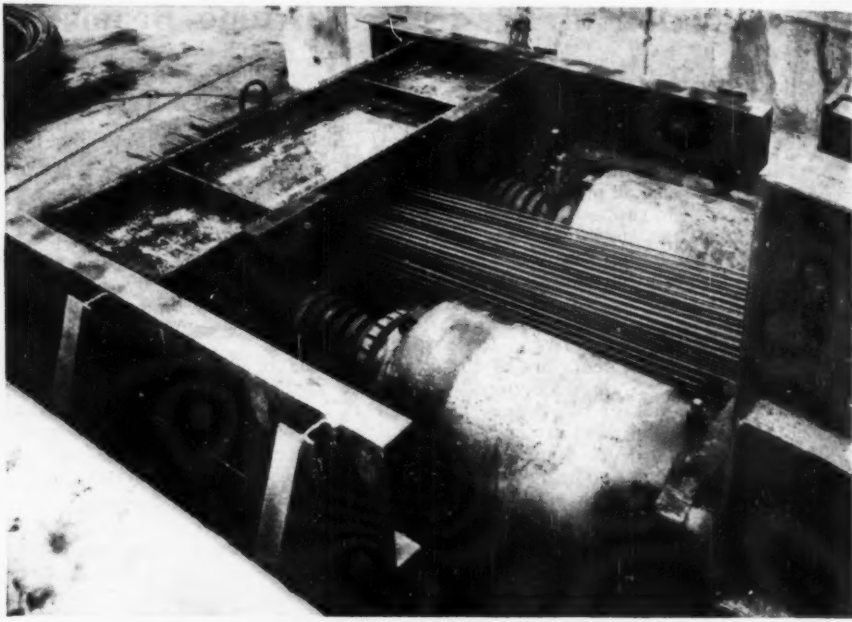


Fig. 2.—A Double Jack.

which is in the foreground of *Fig.* 1. Mild steel binders, secondary reinforcement, and inserts for holes through the web are then fixed, and the timber shuttering for the sides is erected.

Concrete is brought from the mixing plant in a wooden tray of 6 cu. ft. capacity on a fork-lift truck, from which the tray is lifted by a steam-crane which travels on endless tracks on an earthen road alongside the bed. Owing to the stiffness of the concrete it is transferred to the mould by shovels and compacted by pneumatic hammers applied to the outside of the mould and a poker immersion vibrator.

In cold weather the castings are covered and steam is injected under the covers ; the steam is generated in the boiler of the road-roller seen in *Fig.* 1. In inclement or hot weather the bed can be protected for a length up to 75 ft. by means of five corrugated-iron covers, which are each 15 ft. long and span across the bed and travel on rails laid on the outer walls.

When the concrete has matured sufficiently, which is generally seven to ten days after it is cast, the pull on the wires is released by the double jacks shown in *Fig.* 2. The jacks, which are partly screw-type and partly hydraulic, are permanently bolted to steel buttresses on the ends (remote from the tensioning end) of the longitudinal walls and bear against the back of a steel cross-head in which the wires are anchored. During tensioning and while the concrete is maturing the load on the jacks is borne on the protruding threaded shaft and through a serrated lock-ring bearing on the body of the jack. When the pull on the wires is to be relaxed, the load is transferred to the hydraulic part of the jack by rotating the lock-ring, thereby enabling the pull to be released slowly.

The design and construction of the works at Bishops Stortford are under the general direction of Mr. A. K. Terris, M.I.C.E., Chief Civil Engineer of Eastern Region, British Railways. The general contractors are Messrs. W. & C. French, Ltd., who made the beams at their works at Loughton.

Theory and Practice of Making Concrete.

MAJOR R. C. K. MONEY, R.E., writes as follows.

I read with interest the Editorial Note on The Theory and Practice of Making Concrete in your March number, together with extracts from the D.S.I.R. report. They give the impression that mix design is a complete waste of time and results in extra total cost because the resulting mix cannot be placed without great difficulty. Whilst it is true that the highest strengths of concrete must *ipso facto* be difficult to place, because of the limitations of water-cement ratio, this situation is normally met on concrete roads and hardstandings where measures can easily be taken to deal with the difficulties. In building construction working between shutters, with reinforcement in the way, such a mix would be impossible to use.

No mention is made of the fact that mix design starts always with a decision on the workability required to suit the job, the type of mixer, vibrators available, etc. The rest of the calculations for the mix are done from this basis—the final mix design will be the most economical mix for the strength and the workability required using the materials available, though slight adjustment, generally of the water, may still be necessary on the site. The reports make out that the mixes have been designed solely to economise in cement—obviously if workability is not taken into account the mix will be very difficult to place and may, as the examples show, cost more when the job is finished than if the extra cement had been used. In all these cases it is obvious that the mix design was wrong and the work should not have continued. It does not follow at all that mix design is a waste of time and gives no better results than the ordinary rule-of-thumb volumetric mixes, though it must be obvious that it has its place only on the larger job, or where the strength must be very high and the quality carefully controlled. Theory applied without regard to practical difficulties will always result in the tail wagging the dog, as happened in the cases described.

[Our correspondent writes " mix design starts always with a decision on the workability required to suit the job ". In the examples we quoted this was not the case. It is to this subjugation of practical considerations to laboratory tests and unpractical attempts to save cement that we drew attention, and recommended that an experienced engineer on the site should be allowed to vary the quantities of cement and water, while retaining the specified ratio, to produce concrete sufficiently workable for its purpose.

When our correspondent writes " The highest strengths of concrete must *ipso facto* be difficult to place because of the limitations of the water-cement ratio " he expresses the fallacy that is the cause of the unnecessarily expensive concrete sometimes specified by laboratory workers and others. The principle of the control of strength by the water-cement ratio is that the strength is the same whatever the proportion of cement or of water in the concrete so long as the ratio is maintained, and that a more workable mixture is as strong as a stiffer and probably unworkable mixture so long as it contains more cement as well as more water in the specified ratio.

Many stiff mixtures specified by laboratory workers can be made workable by the addition of a plasticiser without much reduction in strength. Also a mixture that may be too stiff and unworkable for reinforced concrete structures may not be unsuitable for plain concrete foundations or other large masses of concrete as in dams.—ED.]

Accidents on Civil Engineering Works.

THE Federation of Civil Engineering Contractors has issued a pocket-size " Safety Guide for Works and Civil Engineering Construction ". The advice given is based on an investigation of 3,000 accidents that have occurred on such works and it is believed that the adoption of its recommendations will result in fewer accidents. Copies can be obtained from the Federation at Romney House, Tufton Street, London, S.W.1, at 1s. each.

Book Reviews.

"Béton Précontraint, Tome II, Constructions Hyperstatiques." By Y. Guyon. (Paris: Editions Eyrolles. 1958. Price 9300 fr.)

THE author of this book has for many years collaborated with M. Freyssinet in the design of some of the most outstanding constructions in prestressed concrete, and his views on prestressed concrete and his writings command respect. His high standards are maintained in this book (printed in the French language), which is the second volume of a comprehensive work on the subject and deals with continuous beams, frames, and some problems of slabs. The book is in two parts; in the first part are given methods of design based on the theory of elasticity, and in the second the non-elastic behaviour of continuous structures is examined. Many tests on large-scale models and some on actual structures are critically described in order to compare their actual and predicted behaviour.

The first part commences with a description of concordant cables, that is an arrangement of the prestressing force in a continuous member so that it does not cause secondary reactions. The theories are applicable to continuous beams, rectangular frames, and multiple-bay structures. One chapter deals with temporary joints for controlling the position or direction of the prestressing force at some stage in construction, and with joints in precast construction. In another chapter the costs of statically determinate and continuous structures are compared. Members with a non-uniform section and those with a varying prestressing force are adequately treated. The application of the theories is demonstrated by examples, many of which are taken from some of the important structures designed by the author.

Having developed methods of design and analysis based on the elastic theory, the author deals in the second part with the actual behaviour of structures, and from the results of many tests has produced a method of calculating the loads causing failure. The description of the tests is not only a record of work that has been done, but the presentation also indicates fruitful fields for future research.

The book is a full exposition of the theories and practice of the subject, giving the information, and indeed some of the author's enthusiasm, necessary for the conception, detailed design, and construction of continuous structures in prestressed concrete.—J. E. G.

"Reinforced Concrete Fundamentals." By P. Ferguson. (London: Chapman & Hall, Ltd. 1958. Price 76s.)

THE "fundamentals" in the title of this students' book are those of the design of reinforced concrete structural members in accordance with practice in the U.S.A. Emphasis is placed on load-factor (or ultimate-load) methods, but comparisons are made with the elastic method. Most of the chapters deal with the common theory and practice, but some secondary features are discussed; in particular the treatment of bond, resistance to shearing, deflection, and a short consideration of torsion, are interesting reading. Instances where the author expresses opinions differing from the code of the American Concrete Institute are also interesting. The subject matter is limited to the design of separate structural members of a building and, except in the case of retaining walls, does not extend to complete structures. The principles of prestressed concrete are explained briefly.

"Linear Structural Analysis." By P. B. Morice. (London: Thames & Hudson, Ltd. 1959. Price 35s.)

THE author describes this book as an introduction to the influence-coefficient method of calculation applied to statically-indeterminate structures. It serves this purpose well, and is a welcome departure from the turgid and non-standard methods of computation which have for too long bedevilled books on structural engineering. Consequently the fundamental simplicity and generality of the principles of elastic analysis are made refreshingly clear. The main headings are definitions and strain energy, influence coefficients, indeterminacy, properties of matrices, computation and scale factors, resultant stress distributions, transformations, release systems, the flexibility matrix, and the use of computers. Some examples are given. The presentation is clear and concise. The

addition of an index would be an advantage. The book is a worth-while addition to the library of an engineer who wishes to keep abreast of modern methods of computing.

The special merit of the book is the systematic application of matrix algebra to a typical engineering problem. Having mastered the method, an engineer is equipped to tackle intelligently a wide range of engineering problems by methods conforming to the requirements for the use of computers. The author warns those who are not familiar with this mathematical expedient that, despite its fundamental simplicity, a "craftsman's" knowledge of it is attained only by constant practice.—A. H. D.

"The Hardening of Concrete Under Winter Concreting Conditions." By A. Nykänen and S. Pihlajavaara. (Helsinki: Valton Teknillinen Tutkimuslaitos Statens Tekniska Forskningsanstalt. No price stated.)

THE chapters are entitled The Structure and Hardening of Concrete, The Fundamentals of the Freezing Phenomenon of Concrete, The Freezing of Concrete, and The Prevention of Frost Damage, and describe the results of investigations made at the Finnish State Institute for Technical Research. Generally the work (which is printed in the English language) is a summary of the results of tests made elsewhere, and details are given of the tests made to confirm existing theories on the subject of concreting in cold weather.

"Ausbeulen, Theorie und Berechnung von Blechen." By Curt F. Kollbrunner and Martin Meister. (Berlin: Springer-Verlag. 1958. Price 42 D.M.)

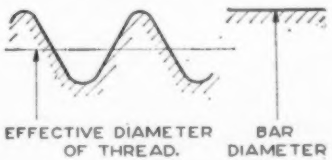
THIS is a survey of the theory and calculation of buckling in thin plates. In addition to solutions based on the direct integration of differential equations, numerical methods are introduced which are claimed to permit the solution of complicated problems with limited mathematics. The greater part of the book deals with rectangular plates. Many combinations of boundary conditions and loading in the elastic and plastic ranges are investigated. The work does not apply directly to concrete slabs reinforced in two directions.

A Congress in Rotterdam.

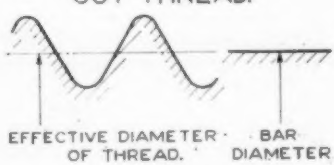
THE International Council for Building Research, Studies, and Documentation is to hold a congress in Rotterdam in September next to discuss the design of structures, standardisation, research on large concrete elements, housing in tropical areas, flat roofs, insulation, and other subjects. Further information may be had from the Building Research Station, Garston, near Watford, Herts, or the Secretariat of the Congress, c/o Bouwcentrum, Postbox 299, Rotterdam, Holland.

Improved Threads on Prestressing Bars.

THE threads now provided on steel-alloy prestressing bars supplied by McCalls Macalloy, Ltd., conform to the profile of the standard unified fine screw-threads (UNF). The threads are formed on the ends of the bar by a cold-rolling process and the bar is subsequently stress-relieved in a propane-oxygen gas-fired furnace. A comparison of the new rolled thread and the previous cut thread is given below. The working tensile stress in the bar is increased from 42 tons to 45 tons per square inch of cross-sectional area of a bar of normal diameter. Since the effectiveness of the anchorage is now independent of the position of the nut, a tolerance of plus or minus 2 in. can be permitted on the length of the concrete member. A smaller coupling is used with the rolled thread. Bars up to 1¼ in. diameter are obtainable.



CUT THREAD.



ROLLED THREAD.

A Plant for Ready-mixed Concrete.

Fig. 1.

A NEW plant for the production of ready-mixed concrete commenced operation in April last in Rochester, Kent. The plant (*Fig.* 1) is capable of producing daily up to 500 cu. yd. of concrete complying with any specification. The aggregates are delivered by river to the wharf on which the plant is installed and loaded by a steam crane into the temporary storage hopper of a conveyor. The conveyor, which is inclined at an angle of 38 deg., has two continuous belts moving one above the other at the same speed, and with two adjacent faces moving in the same direction. The aggregate is contained between them, lying on the concave lower belt and being prevented from slipping downwardly by paddles projecting from the underside of the upper belt. The edges of the two belts touch, so that the aggregate cannot spill out at the sides.

Some 200 tons of aggregate can be delivered hourly to the main storage bin, which has a total capacity of 120 cu. yd. in eight compartments and is at the top of the mixing plant. The cement is stored loose in an underground bin of 50 tons capacity and is conveyed pneumatically to a bucket-elevator which carries it to the top of the structure where it is blown through cotton filters into a compartment in the main storage bin.

If the concrete is not to be mixed at the plant the dry materials are weighed and delivered through a chute to truck mixers, water being added and mixing started when approaching, or on arriving at, the site. If the concrete is to be mixed in the plant, the water, cement, and aggregate are measured by weight and passed into the mixer, which has a capacity of 3 cu. yd. After mixing for two minutes the drum is tilted and the concrete is delivered through a chute into a lorry which agitates the mixture while delivering it to a site within a radius of about 15 miles. The weighing, mixing, and tilting are all controlled pneumatically by one man.

The plant was designed and built for the Brice Pre-mixed Concrete Co. (Kent), Ltd., by Winget, Ltd., of Rochester.

Films and Models of Civil Engineering Works.

A LIST of films and models that are available for loan to educational authorities, professional associations, and others interested in civil engineering has been prepared by the Federation of Civil Engineering Contractors, Romney House, Tufton Street, London, S.W.1, from whom copies may be had free of charge.

Sliding Wall Slabs into Position.

THE accompanying illustrations show the method used in placing precast wall slabs in a building partly two stories and partly three stories high and measuring 125 ft. by 440 ft. in plan. The building is at Akron, Ohio, U.S.A., and the following is abstracted from "Engineering News-Record".

A cross section of a slab, which is 7 ft. 6 in. wide by 11 ft. 6 in. high, is shown in *Fig.* 1. These were cast on the ground in stacks, the window opening being formed by a wooden shutter. When the columns and the floor and roof slabs (which project beyond the face of the building) were erected, a track formed of inverted steel channel was welded to anchors cast on the tops of the floor slabs and pairs of steel angles were bolted to the undersides of the upper floor and the roof to form a guide and support for the tops of the slabs.

A steel-faced groove was cast in the bottom edge of the slabs, which were lifted on to the track (*Fig.* 2) and pushed along to the required position. It was found that if the track were greased the slabs could be easily pushed along by hand. Part of the completed building is shown in *Fig.* 3.

By this means 30,000 sq. ft. of wall panels were placed in position in ten days. The slabs were made and erected by the Marietta Concrete Corporation.

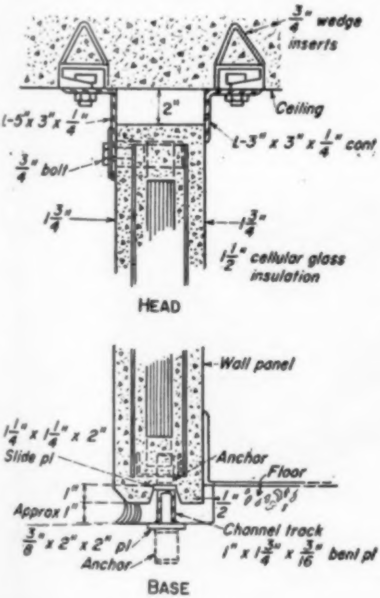


Fig. 1.

Fig. 2.

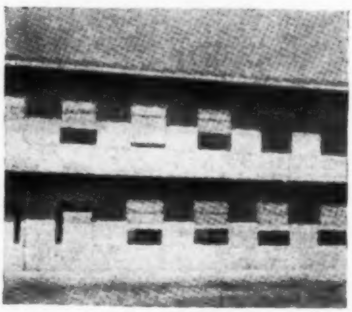


Fig. 3.

FIFTY YEARS AGO.

From "CONCRETE AND CONSTRUCTIONAL ENGINEERING", April–May, 1909.*

FROM A SPECIFICATION.—Contractors will be required to obtain their Portland cement from firms of repute, and it must conform with the British Standard Specification and also comply with a special test for soundness. Added gypsum will not be allowed. Concrete must be mixed in the proportions of (for columns, beams, floors, and the like) $4\frac{1}{2}$ cub. ft. of sand to one bag† of cement and 9 cub. ft. of aggregate. For work in tidal waters and all piles—one bag of cement, $3\frac{3}{4}$ cub. ft. of sand, and $7\frac{1}{2}$ cub. ft. of aggregate. The steel, in the form of round bars and strip, is to be mild steel produced by the open-hearth basic or acid process; neither Bessemer nor high-carbon steel will be allowed. Tests are to be conducted upon any secondary beam in any floor or roof panels selected by the engineer or architect, and, when under full test load, no part of the construction must develop a proportionate deflection greater than 1-600th of the span.

* "Concrete and Constructional Engineering" appeared in alternate months until September, 1909.
† A bag of cement weighed 224 lb. and was carried on a man's shoulders.

Conference on "Shell" Roof Construction.

A CONFERENCE is to be held in Madrid from September 16 to 19, 1959, to discuss the design and construction of "shell" roofs of reinforced and prestressed concrete and of other materials. Further information can be obtained from the Shell Colloquium, Laboratorio Central de Ensayo de Materiales de Construccion, Alfonso XII, 3, Madrid. Applications for registration should be made before May 15.

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